Design and Construction of Interstate 10 Drainage Tunnels in Phoenix

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Drainage of the depressed Interstate 10 inner-loop extension in Phoenix, Arizona, required the construction of a conveyance system composed of pressure tunnels, 14 to 21 ft in diameter, beneath the city. The geologic and engineering parameters specific to this project and the method of construction used in stream-transported alluvial soils are described. Minimization of surface settlement and concern for lowering of the groundwater table, which is almost completely controlled by the nearby Salt River, influenced construction methods and contractual provisions.

Interstate 10, a major link between East and West, runs from Florida to California, passing through the southern tier of states. I-10 is complete except for a small portion in the Phoenix area. This last link of I-10 will pass through the city to form an inner loop through the downtown business district. Conceived as a depressed roadway, the sunken section dividing the watershed into two regions would intercept the surface drainage of the Phoenix basin. Several times a year the Phoenix area is prone to flash flooding. The runoff from these storms moves from the mountains in the North to the Salt River. To prevent inundation of the depressed roadway and flooding upstream, runoff storm water collected from the northern half of the drainage area will be conveyed in a series of pressure tunnels (siphons) to the Salt River. The tunnel system is shown in Figure 1, and the prominent features of each of the tunnels are given in Table 1.

The geologic features unique to the Phoenix area and the design and construction of the tunnels are described. Details of construction methods, equipment, and surface effects of tunneling are also included.

GEOLOGY

The project area, in the desert region of the Basin and Range Physiographic Province, is characterized by fault block mountains with intervening basins filled with deep deposits of unconsolidated sediments shed from the mountains. The topography is nearly flat and slopes at less than 1 percent to the southwest toward the Salt River. Most flows in the Salt River are controlled by a series of upstream dams operated by the Salt River Project. Release from these dams, combined with the natural drainage from below the dams, has reached peak flows of as much as 170,000 ft^3/sec in past years. However, the riverbeds are normally dry.

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The typical soil profile in the Phoenix Basin consists of stream-transported, alluvial, coarse-grained, poorly graded, "river run" sediments overlain by fine-grained sediments deposited in areas isolated from the main stream channels. In the Phoenix project area, the fine-grained sediments are mostly sandy clays of depths ranging from 10 to 30 ft. These sediments are underlain by sand, gravel, and cobble deposits of considerable depth. Groundwater readings obtained from borings in the project area indicate that the water table is at depths ranging from 60 to 95 ft below the ground surface.

The overlying sandy clay material can be described as light brown to brown in color and of medium plasticity (classified as CL material according to the Unified Soil Classification System). The deposit has light to moderate calcite cementation, which generally increases with depth. Dry density (γ_d) of the deposit ranges from 90 to 115 lb/ft^3, with an average value of 105 lb/ft^3. Moisture contents of the soil are generally below the plastic limit and range from 6 to 20 percent, with an average value of 12 percent. In general the sandy clay has an internal angle of friction (ϕ), ranging from 22 to 48 degrees, with an average value of 30 degrees. Similarly, the shear strength of the material is good; the cohesion intercept ranges from 0.25 to 0.9 ton/ft^2, with an average value of 0.4 ton/ft^2. It should be noted that the shear strength of the material was tested in direct shear after samples had been saturated, consolidated, and then sheared under undrained conditions.

Located below the fine-grained sandy clays is a thick deposit of sand, gravel, and cobbles (SGC). This SGC deposit is grayish brown in color and roughly stratified. Grain sizes of the deposit range from trace fines to boulders as large as 18 in. in diameter. Most of the materials that make up the SGC deposit are subrounded sands and gravels, classified as GP material according to the Unified Soil Classification System. The SGC deposit has a high relative density with a dry density (γ_d) of approximately 140 lb/ft^3. The internal angle of friction (ϕ), of the uncemented material is 40 degrees and perhaps as high as 50 degrees for the cemented material. Predominant rock types in the SGC deposit include sandstone, quartzite, and granite. A generalized soil profile with soil description and representative soil parameters is shown in Figure 2.

GEOTECHNICAL INVESTIGATIONS

A limited conventional subsurface exploration and testing program was undertaken to provide information on subsoil and groundwater along the project corridors and to verify

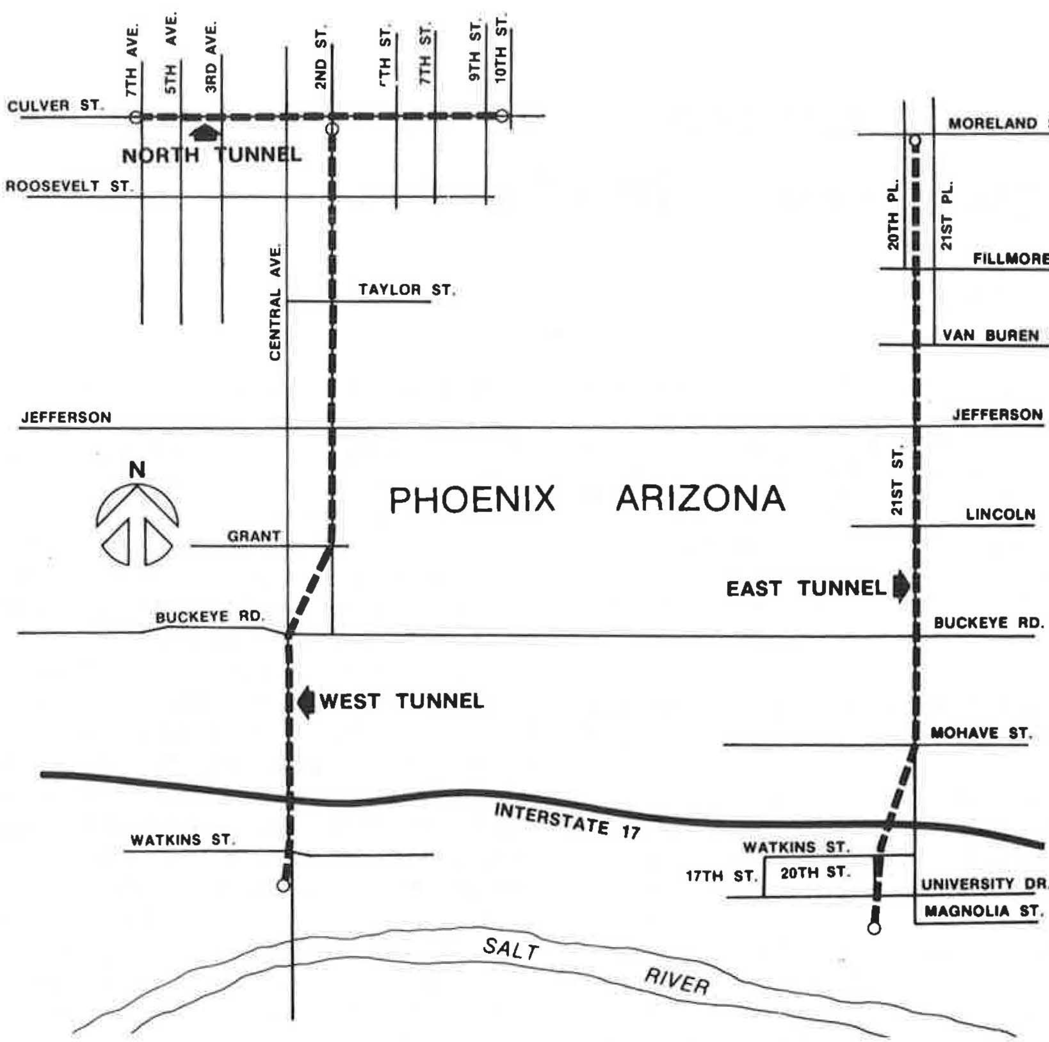

FIGURE 1 Site location.

TABLE 1 FEATURES OF I-10, PAPAGO FREEWAY, DRAIN TUNNEL SYSTEM

Tunnel and Identification	Finished Diameter (ft)	Total Length (ft)	Excavated Diameter	Total Excavated Material (yd^3)	Total No. of Segments	Finish Concrete (yd^3)	Design Flow (ft^3/sec)
North tunnel I-ID-10-3(187)	14	6,554	17 ft	56,000	6,655	11,000	2,200
East tunnel I-ID-10-3(188)	21	13,542	24 ft 10 in.	243,000	13,542	31,000	5,000
West tunnel I-ID-10-3(189)	21	13,680	24 ft 10 in.	245,000	13,680	32,000	5,000

observations and data uncovered by a literature search. Small-diameter borings were drilled to depths of 100± ft below existing grade; borings were drilled with a Becker hammer drill and advanced with the ODEX (overburden drilling with the eccentric method system). Field permeability tests were performed in selected borings at three locations.

During the prebid phase of this project, prospective bidders had the opportunity to inspect the ground within the project area from four large-diameter observation holes. These holes were drilled using a 36-in.-diameter helical auger to a depth of approximately 60 ft. A 36-in.-diameter steel casing with observation ports placed approximately every 5 ft below a depth of 20 ft was installed to provide safe entry. More than 40 prospective bidders and consultants were lowered into each of these holes (Figure 3).

EXPECTED GROUND BEHAVIOR

The behavior of the ground at tunnel level was expected to vary primarily as a function of the amount and extent of cementation encountered within the SGC deposits in which all of the tunneling was to occur. It was anticipated that firm, raveling to running ground conditions would occur at the tunneled face. These conditions might occur singly or in combination within the length of a single advance because of the erratic and variable nature of the river-deposited SGC materials.

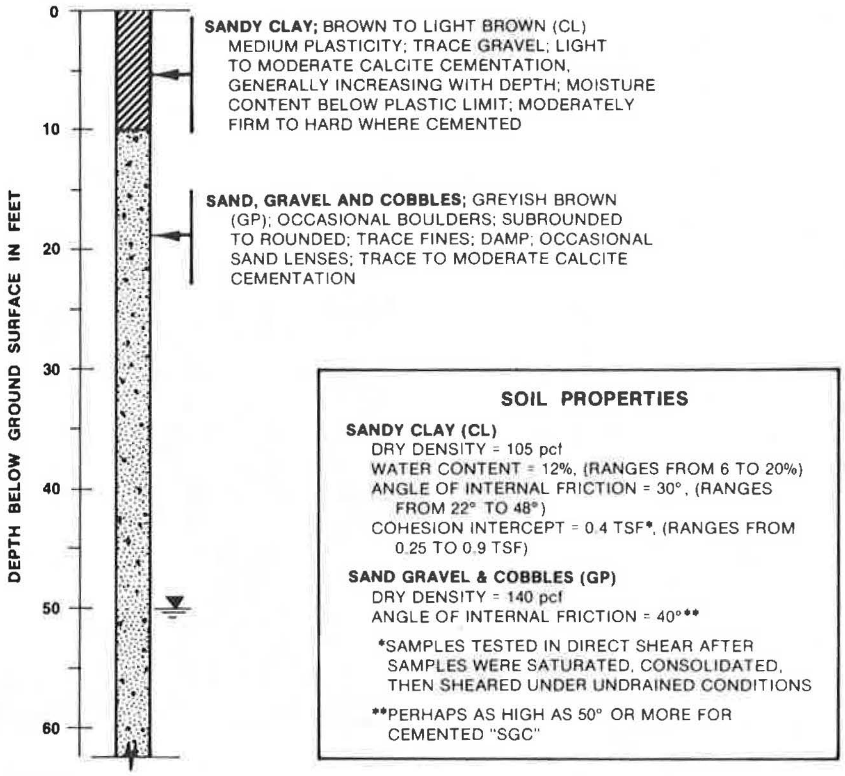

FIGURE 2 Generalized profile and material properties.

FIGURE 3 Inspection or large-diameter exploratory boring.

The SGC, when uncemented and free of apparent moisture, would exhibit behavior of almost cohesionless granular soil through which tunneling could be carried out only with complete protection of the face tops and sides of the excavation. Where the material was dense and cemented, tunneling was expected to be performed with little difficulty and with negligibly small settlements, except where raveling or runs at the face occurred. Without adequate face support, such runs may occur with attendant large and irregular loss of ground. The consequence may be runs that completely invade the heading, leading to the formation of a sinkhole at the surface of the ground.

CONTRACT PROVISIONS AND DESIGN

The potential variability in ground stability and the necessity of restricting settlements throughout the project area to minimize tunneling effects on overlying utilities and surrounding foundations dictated that the contractor be allowed maximum flexibility in choosing techniques and methods. At the same time, sufficient performance criteria were necessary for the owner to adequately control the work. To provide this control and flexibility, the contract specifications were written as broadly as possible. Although all specifications contain some prescriptive language, these specifications also contained performance or minimum design criteria. All contractors were required to prequalify, both financially and technically, before building.

Contractor provisions included

- Required use of a shield and tunnel-boring machine;
- Alternative design of initial support systems subject to minimum design criteria: (a) sustain full overburden but no less than 6,000 lb/ft^2, (b) sustain a diametrical distortion of 0.5 percent, (c) include provision for all loads including construction-induced load, and (d) maximum rib spacing or segment length of 5 ft 0 in.;
- Contractor responsible for maintenance and protection of existing overlying utilities and foundations;
- Contractor responsible for settlements in excess of 1½ in.;

- Force account for roadway and sidewalk repair (owner assumed responsibility for damage to residential structures);
- Alternative designs for final lining subject to the minimum criteria;
- Liquidated damage of $4,000/per day;
- Lowering the groundwater table to obtain workable conditions;
- Detailed preconstruction survey of area; and
- Escrow bid documents.

The interrelationship among excavation equipment, sequence of construction, and personnel safety dictated that the contractor be responsible for the design of the initial support system and tunnel shield. The contract required that the tunneling shield have a well-proportioned hood and be equipped with poling plates, breast tables, boards, or similar mechanisms to ensure positive control of the face of the excavation at all times and minimize surface disturbance and related soil movements.

The contract documents contained two alternative initial support systems, a precast concrete segmental lining and structural steel rib and wood lagging. After the initial support system was installed, a final cast-in-place lining was to be installed.

CONSTRUCTION PROGRESS AND OBSERVED CONDITION

The three tunnels were bid as a single contract package to enable common use of tunnel excavator, forms, and other construction materials. The tunnel project was first advertised in February 1984. Bids were received on May 10, 1984. A tabulation of bids is given in Table 2.

TABLE 2 BID RESULTS

Amount ($)	Contractor
49,633,450.00	Shank-Artukovich-Ohbayashi-Gumi
51,827,900.00	J. F. Shea Co., Inc.
55,197,920.00	Schiavone Construction Co.
55,852,401.00	Kiewit Construction Company and Kiewit-Grow, a joint venture
57,347,170.00	Phillips and Jordan, Inc., Dick Corporation, and Paschen Contractors, Inc., a joint venture
61,077,000.00	S & M-Mancini-Greenfield, a joint venture
63,441,932.00	Loram-Mergentime, a joint venture
64,291,389.00	Harrison Western Corporation
64,587,933.07	S. A. Healy Company
64,895,160.00	Traylor Bros., Inc.
65,556,110.00	Dillingham Construction, Inc., Ball, Ball & Brosamer, Inc., and John Mowlem & Company, PlC, a joint venture

The low bidder, Shank-Artukovich-Ohbayashi-Gumi, joint venture, was awarded the contract on June 1, 1984, and began sinking the construction shaft for the east tunnel in September 1984. In November 1984, three large-diameter wells were installed about the shaft to control local groundwater. An additional eight wells were installed along the alignment. Drawdowns of as much as 95 ft were observed. In general, groundwater was not a problem during tunnel excavation. Tunnels were excavated using a semimechanized shielded excavator, equipped with radial orange-peel doors and a bucket-type excavator, manufactured by the Hitachi-Zosen Corporation of Osaka, Japan (Figure 4). Critical to timely completion of the east tunnel was early delivery of the tunnel shield. The Japanese-manufactured shields were delivered in 6 months and were assembled on site. Generalized machine specifications are given in Table 3. The contractor proposed the use of a nonexpanded precast concrete segmental lining system. The segments for the 21-ft-diameter tunnel were 10 in. thick, and each ring was composed of four segments and a key. An 8-in.-thick

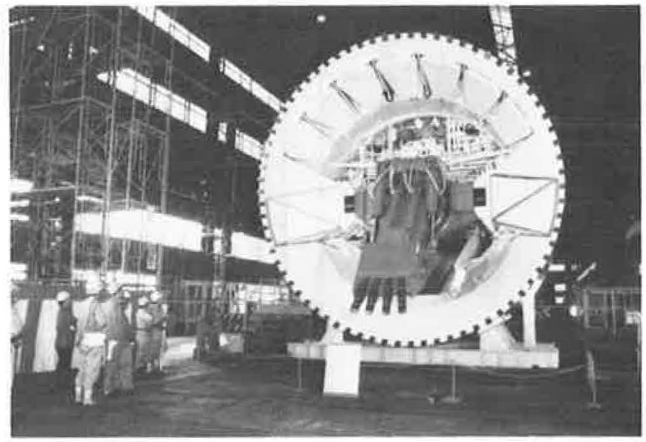

FIGURE 4 Shield 24 ft 10 in. in diameter.

TABLE 3 TUNNEL SHIELD AND INITIAL SUPPORT CHARACTERISTICS

Item	14 ft 0 in. Tunnel	21 ft 0 in. Tunnel
Nominal ring outside diameter (D_s)	16 ft 8 in.	24 ft 6 in.
Nominal segment thickness (h)	8 in.	10 in.
Width of segment (W)	4 ft 0 in.	4 ft 0 in.
Number of segments per ring	4 + key	4 + key
Approximate weight		
2 below spring line (each)	4.9 kips	9.1 kips
2 above spring line (each)	4.9 kips	9.1 kips
1 crown key	0.4 kips	0.7 kips
Reinforcement		
Longitudinal	W1.4 WWF, EF @ 12 in. on center	W2.9 WWF, EF @ 16 in.
Transverse	W3.5 WWF, EF @ 4 in. on center	W5 WWF, EF @ 3 in.
Area of steel (A_s)	0.105 in.^{2}/ft/face	0.2 in.^{2}/ft/face
Reinforcement ratio	0.012	0.018
Shield outside diameter (D_o)	16 ft 11.5 in.	24 ft 10 in.
Tail skin clearance	1 in.	1 in.
Tail skin thickness (t)	1 in.	3/4 in.
Ratio $D_o - 2t/D_s$ =	1.0068	1.0099
Percent excess excavation	2.7	3.5
Shield propulsion system		
Number of jacks	18	18
Thrust force per jack	441 kips	221 kips
Face breasting system		
Number of jacks	7	7
Thrust force per jack	66 kips	66 kips

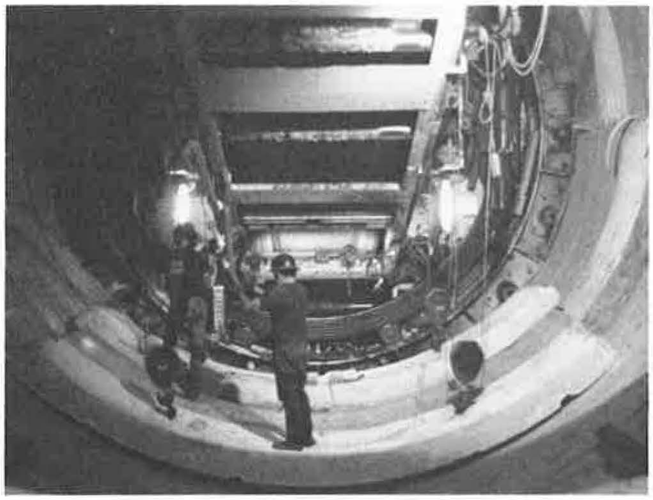

FIGURE 5 Segment erection.

precast segmental lining of similar geometry was proposed for the 14-ft-diameter tunnel. Figure 5 shows segment erection in the tail of the shield 24 ft 10 in. in diameter.

The contractor began tunneling on February 1, 1985, at a previously constructed 50-ft-diameter shaft at the east tunnel. Approximately 1 month later, tunneling was suspended when the contractor determined that the tunneling shield was not capable of advancing.

The material at the face was generally described as running sands, cobbles, and gravel. Persistent runs and chimneys to the surface had formed after each shove. The contractor was unable to control losses of ground at the face—it appeared that the machine was not capable of exerting any pressure on the face and that the excavator and breasting doors limited access to the face. The mining method used by the contractor generally contributed to this condition. He would excavate a central plug of ground and then shove ahead. This displacement allowed soil movement into the centrally excavated plug, initiating massive soil movements. To allow continued advancement, the radial face doors were relieved and allowed to rotate inward, fully relieving the entire soil face. Loss of this face support enabled ground movements to extend behind the region supported by the hood of the shield, resulting in subsequent surface subsidence and chimneys.

Subsequently, the contractor modified his tunneling equipment, as shown in Figure 6, to include the addition of poling plates, breasting boards, and a breasting table to his shield to increase the lead or hood of his machine. The poling plates and other face control mechanisms, when properly advanced, generally appeared to have improved the contractor's ability to minimize ground movements and stabilize the excavated face.

RATES OF PROGRESS

After the machine modifications, tunneling began in earnest on the east tunnel; the average rate of advance was approximately 75 ft/day. Two 9½-hr production shifts and one 5-hr maintenance shift were employed. During tunneling, several 1,000-yd^3 or larger sinkholes developed above the tunnel and were filled before tunneling was resumed.

These large surface movements were tolerable because the area above the tunnel was cleared right-of-way. Holing through of the east tunnel was completed in March 1986, as shown in Figure 7. Shortly thereafter, concrete lining was begun. Concrete was placed by conventional tunneling techniques: telescoping metal forms were used and placement was done on a continuing three-shift basis—pour, set steel and forms, strip and move ahead (Figures 7–9). The tunneling shield was moved to the west tunnel and rebuilt. The shield and equipment were modified, including installation of larger cutting teeth on the poling plates and an increase in the overcutter. Tunneling progress on the west tunnel, now under way, is approximately

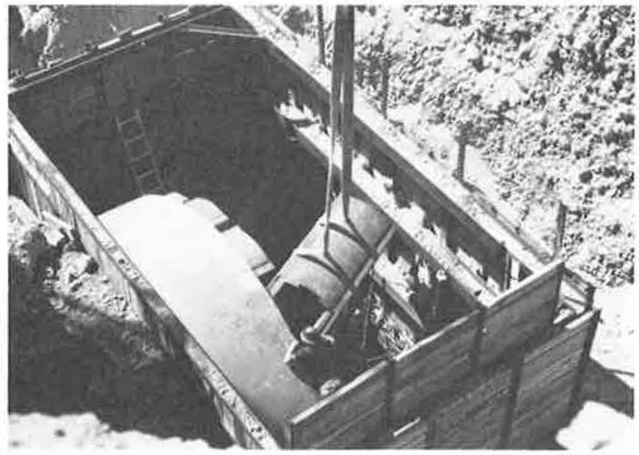

FIGURE 6 Installation of poling plates.

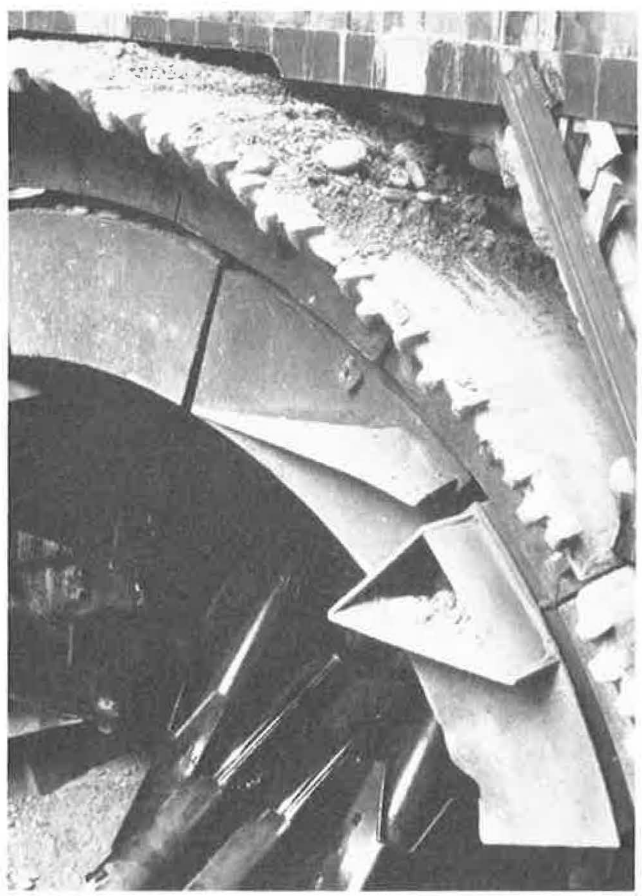

FIGURE 7 Holing through.

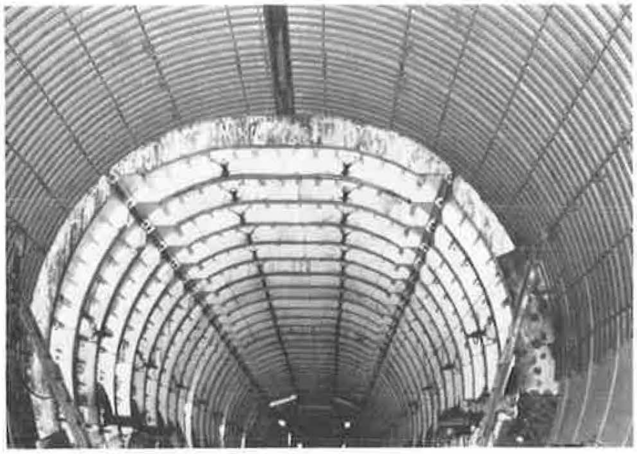

FIGURE 8 Final lining placement.

FIGURE 9 Finished tunnel.

140 linear feet per day. To date, progress on the best day was 228 linear feet.

MITIGATION OF GROUND LOSSES

The large ground losses created by construction of the east tunnel caused considerable concern. Because the west tunnel was to run beneath heavily traveled Central and Second Avenues in the middle of downtown Phoenix, no such losses could be tolerated. To minimize the induced settlements and provide a quick response mechanism if a large ground loss did occur, a compaction-consolidation grouting scheme was developed.

Compaction grout holes are drilled from the surface in advance of tunneling. As tunneling proceeds, and the shield passes each hole, as required, a pump crew feeds a low-slump soil and cement mixture into the ground above the tunnel envelope until refusal or until deflection of the segmental liner occurs. If a serious run occurs, large amounts of grout can be pumped through a series of neighboring holes. At several locations, chemical grouting was performed in advance of excavation to forestall ground losses. Ground settlements are generally kept to minimal levels, on the order of $^1/_4$ to $^1/_2$ in.

ACKNOWLEDGMENTS

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